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DYNAMIC EFFECTS ON CULVERTS FOR HIGH SPEED TRAINS

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Abstract *Culverts are very common in recent railway lines. Wild life corridors and drainage conducts often fall in this category of partially buried structures. Their dynamic behavior has received far less attention than other structures such as bridges but its large number makes that study an interesting challenge from the point of view of safety and savings. In this paper a complete study of a culvert, including on-site measurements as well as numerical modelling, will be presented.*

The structure belongs to the high speed railway line linking Segovia and Valladolid, in Spain. The line was opened to traffic in 2004. Its dimensions (3x3m) are the most frequent along the line. Other factors such as reduced overburden (0.6m) and an almost right angle with the track axis make it an interesting example to extract generalized conclusions.

On site measurements have been performed in the structure recording the dynamic response at selected points of the structure during the passage of high speed trains at speeds ranging between 200 and 300km/h.

The measurements by themselves provide a good insight into the main features of the dynamic behaviour of the structure. A 3D finite element model of the structure, representing its key features was also studied as it allows further understanding of the dynamic response to the train loads . In the paper the discrepancies between predicted and measured vibration levels will be analyzed and some advices on numerical modelling will be proposed.

1. INTRODUCTION

Culverts and underpasses are very common in recent railway lines, including high speed railway lines (HSRL). Wild life corridors and drainage conducts often fall in this category of partially buried structures. According to Eurocode, they are not simple structures for which the impact factor may be obtained using simplified formulas. Therefore, the amount of analysis required in their design is similar to the analysis associated to non conventional bridges, while their cost should be orders of magnitude smaller.

This lack of consistency between cost and design effort is being addressed by Spanish Public Works Office. As a first step, the dynamic response of a set of real structures in the HSRL linking Segovia and Valladolid has been monitored. A preliminary set of the results of these work has already been presented [1], [2]. This paper deals with one of these structures, which dimensions have been found to be very frequent. The first part of this paper presents both the structure and the layout set for the monitoring. Also, the most important features of the recordings obtained during the passage of a series of high-speed trains will be summarized.

In order to gain insight into the dynamic behavior, the structure has been modeled using finite element (FE) techniques. A set of two FE models, one representing the superstructure, the second representing the structure, the embankment and a portion of the soil, will allow performing simulations resulting in time history estimations of some response parameters. All issues regarding FE models, which are different from the ones formerly used by some of the authors [3], will be covered in the second part of the paper.

2. STRUCTURE PRESENTATION AND LAYOUT

The structure selected for monitoring has a square cross section. Its inner opening is 3m wide, and the walls are 0.2m thick. It is build up of 8 precast blocks, each of them being 2m long. Therefore, the total length of the structure is 16m. The mean distance between the ballast base and the upper plane of the structure is 0.6m. The geotechnical information available indicates that the soil is granular, with medium density. The number of blows in standard SPT test varies almost linearly from 10 at 1m depth to 90 at 5m.

Statistical analysis of the inventory of all culverts and underpasses in the HSRL linking Segovia and Valladolid revealed that this dimensions were the most commonly found. Three other factors lead to the choice of the structure. First, it is almost perpendicular to the track axis. Second, its reduced overburden. Third, the topological conditions of the surroundings that allowed both suitable access and good visibility of trains using the line.

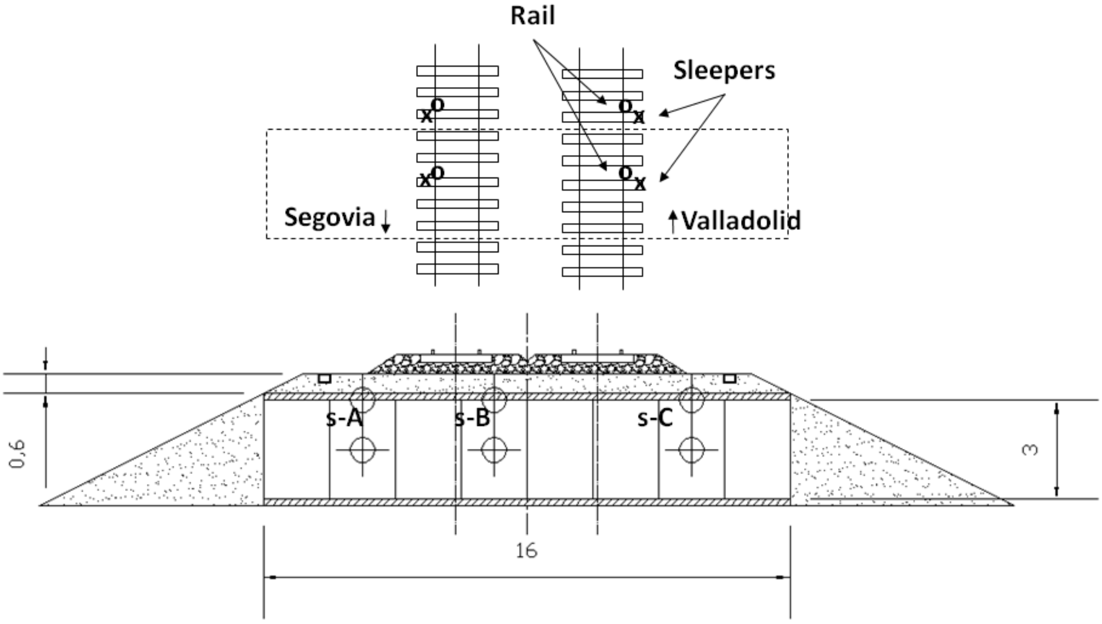

Figure 1. Structure's geometry and sensor positioning.

Both the structure and superstructure have been monitored with piezoelectric accelerometers. In the structure, 6 sensors have been placed in the middle section of three different precast blocks. In each position, a sensor has been positioned vertically at midpoint of the roof, and another horizontally at mid-height of the wall. The relative distances between sensors and track axis are shown in Figure 1.

In the superstructure, sensors have been placed vertically on sleepers (close to the exterior rail-pads) and on the rail foot (at mid-span between two sleepers). Four couple of sleeper and rail sensors have been placed, two on each track. In each case, one sleeper is placed approximately over the underpass mid-span, and the other one is three sleepers further to the north (Valladolid).

In this document, the focus will be on the dynamic behavior of the underpass roof, and only measurements from three sensors (A, B and C in Figure 1) will be taken into consideration.

3. MONITORING RESULTS SUMMARY

The structure was monitored for slightly more than 24 hours. During that time, 46 trains travelled the line. Four different passenger trains were operating in the line. Their nominal axis loads and distances are presented in Figure 2.

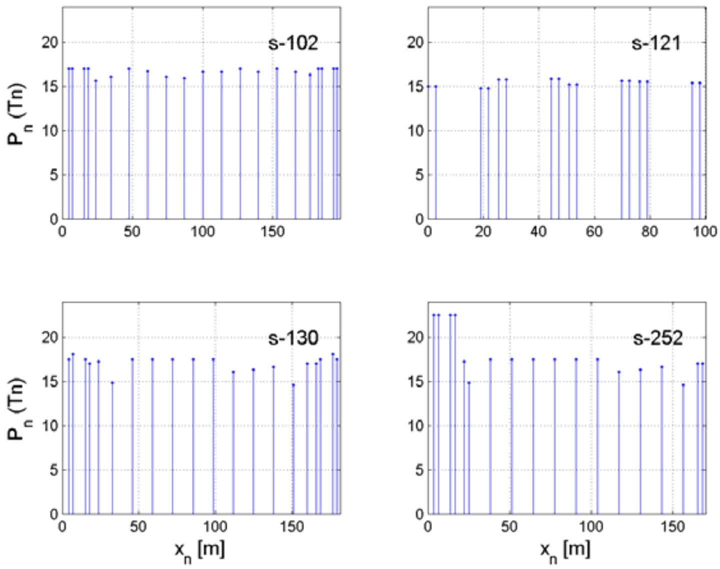

Figure 2. Characterization of trains travelling the line.

A sample of recording obtained is shown in Figure 3. It was obtained during the passage of a s-130 train, which is the RENFE code for Talgo 250, with an estimated speed of 200km/h. Speeds have been estimated in two ways. First, from video recordings of the trains during the tests. Second, measuring time lapses between peaks associated with each axis in root mean square (RMS) acceleration records.

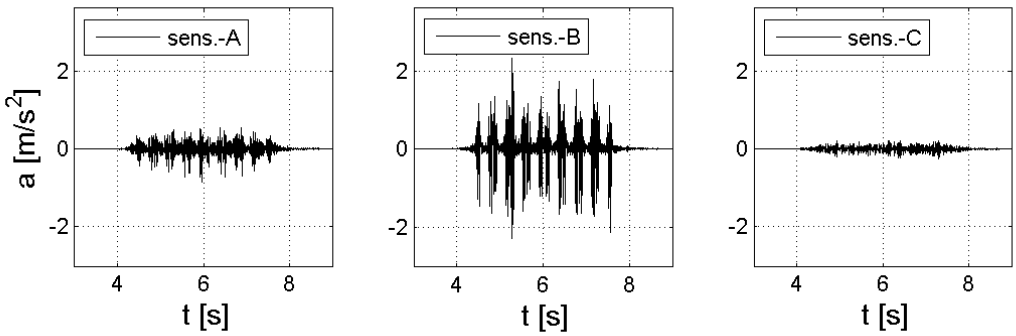

Figure 3. Sample acceleration recording obtained.

Acceleration records show important differences in measured levels depending on the sensor (Figure 4). In this case, the biggest response has been measured close to the center of the structure. Spectral analysis of this signal shows that energy distributes in a wide range between 0 and 120Hz. Analysis of the decay portion of the record indicate a free vibration frequency of about 40Hz. This is interesting because, EC0-A2 [4] indicates that peak acceleration levels must be calculated considering frequencies below the highest value among 30Hz, or1.5 times the fundamental mode and the third vibration mode. In this case it is difficult to establish de vibration modes of the ensemble structure - embankment. However it is clear that there is an important amount of energy far above 1.5*40=60Hz.

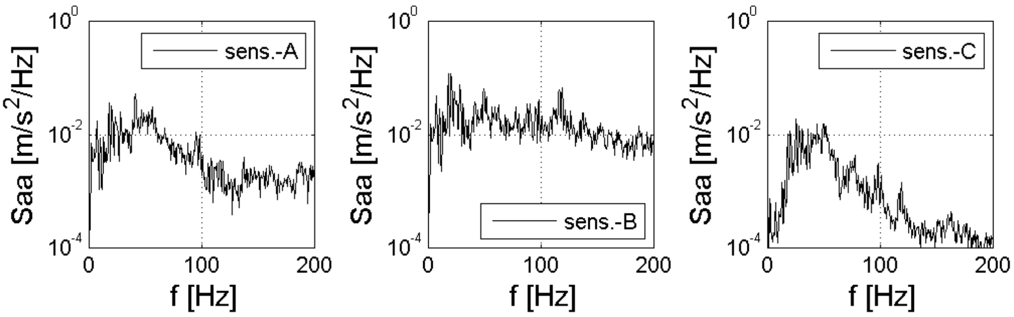

Figure 4. Spectral density from a sample acceleration recorded at the roof of the culvert.

Figure 5 shows an overview of all measurements performed. It shows, for each sensor, the peak acceleration value as a function of train speed. Different symbols (circles, squares, stars and diamonds) have been used for each train type. Also, different colors have been used for each sense (track): from Valladolid to Segovia or from Segovia to Valladolid.

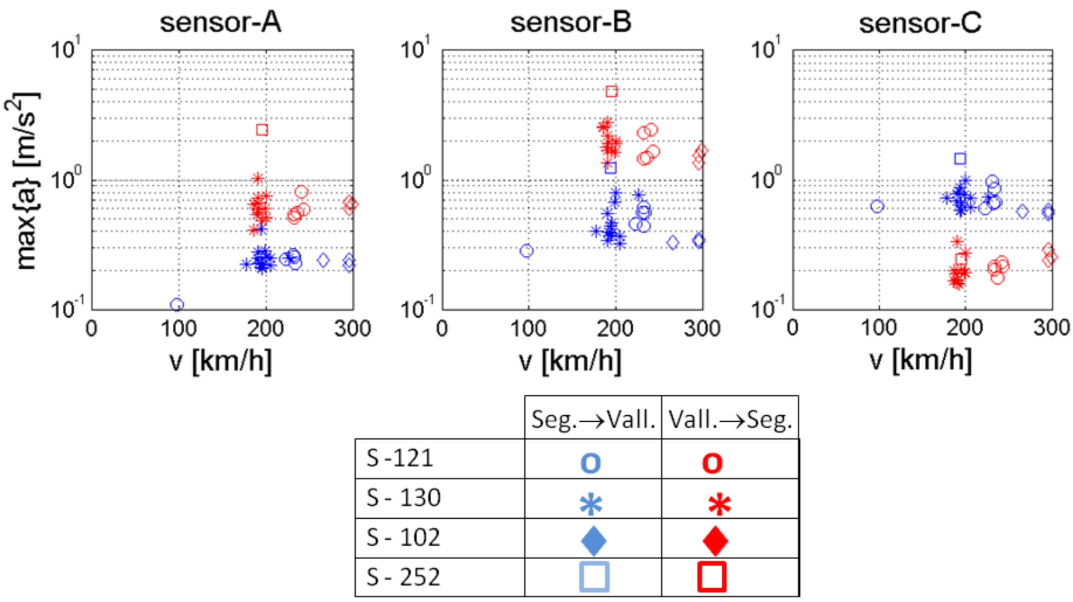

Figure 5. Peak acceleration for each sensor.

It is interesting to notice that speeds for a given train type are in almost all cases very similar.

This is logical considering trains operate almost always at nominal speed. Considering each group of similar samples (a given train at a given speed) the scatter in peak acceleration values seems important. It is also clear that track, that is circulation sense, is important in the response level. As it will be proven, relative distance between sensor and track explain this differences.

Depending on the sense of circulation, relative distances between sensor and circulating track are different. Measurements obtained from both senses of circulation can be considered together if this relative distance is considered. As shown in Figure 6, considering relative distances, the number of available positions increases since the central sensor is not symmetrically placed.

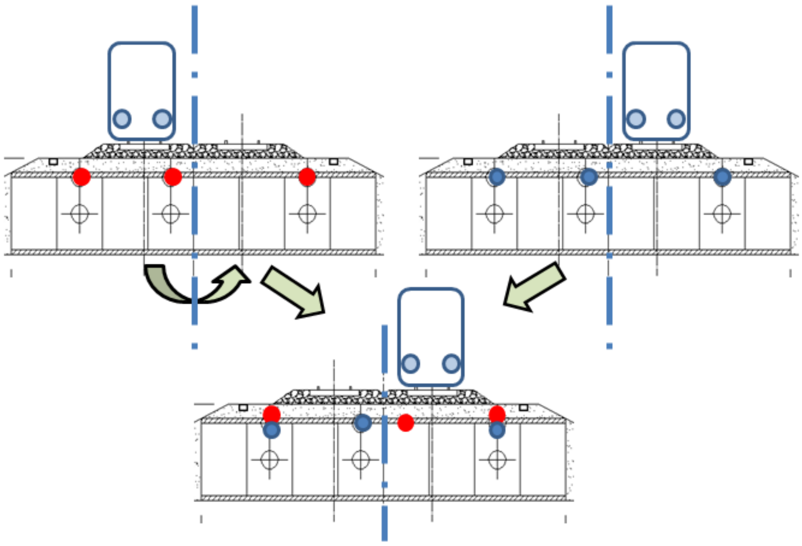

Figure 6. Identification of relative positions of sensors from track axis.

Figure 7 presents a plot of peak vertical acceleration as a function of relative sensor position, along the roof of the underpass. In order to reduce all other sources of scatter, only s-130 trains at around 200km/h have been considered. Each sense of circulation has been plotted in different colors, consistent with the coding in Figure 5. From the figure, it is clear that, at positions ±5m, both senses of circulation lead to similar acceleration levels. Thus proving the importance of relative position in explaining response values.

Figure 7 is interesting because it proves the unequal distribution of acceleration response along the middle plane of the underpass axis. It is clear that demand is much greater below the circulating track than in the opposite track. Next section describes the FE models built in order to have a more detailed description of response levels along the roof.

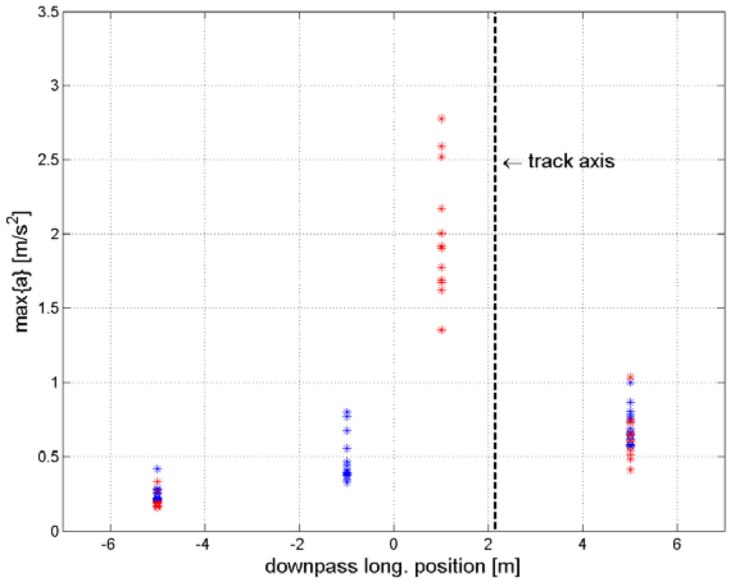

Figure 7. Peak acceleration as a function of relative position along the roof transverse axis.
(s-130 trains at around 200km/h).

4. FINITE ELEMENT MODELS

Vibration levels at different points of the structure will be predicted using a two step method. First, for a given train and a given speed, reaction force records in the ballast-subballast interface will be evaluated at discrete points along the track axis. These points will be considered as source terms in the second calculation step, where the acceleration record at the point of interest is obtained. This is performed by considering as many transfer functions as source points considered.

The first step is performed using a finite element model representing half the track superstructure. In this model the rail is modeled using beam elements. Sleepers are modeled as lumped masses. Spring dashpot elements are used to model both the fastening system and the ballast-embankment ensemble.

A lumped mass representing the unsprung mass of half an axle is set to move at the train’s speed. A spring dashpot element is used to model contact. A moving load representing axle load is linked to this mass. Time integration of the motion equation allows obtaining reaction force histories at discrete points of the ballast sub-ballast interface. Since all considered elements are linear, time histories for different axle loads can be obtained by scaling.

Moreover, considering axle distances and train velocity, time lags can be computed making it possible to build the complete history of reaction forces when the train passes.

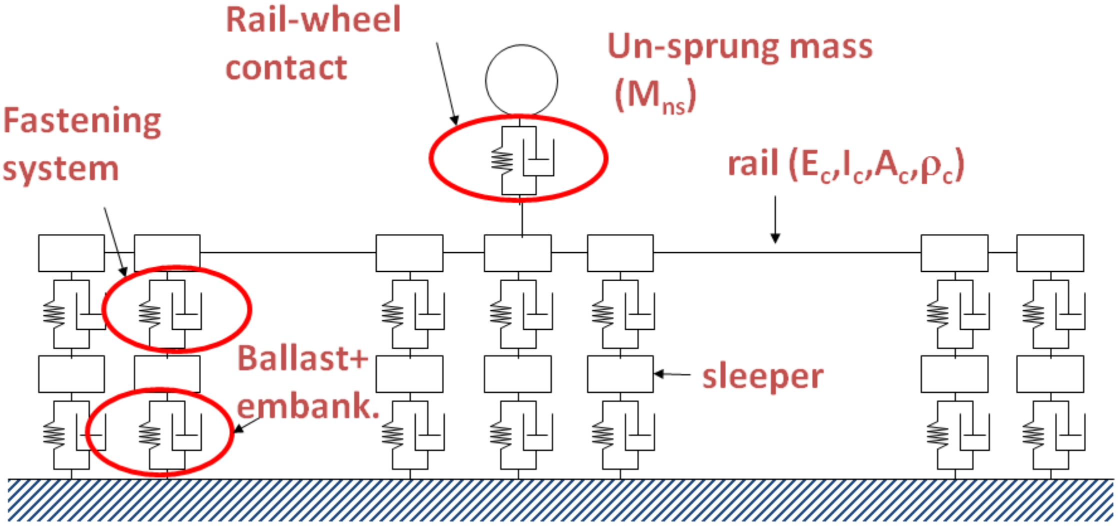

Figure 8. Finite element model used for obtaining reaction forces at interface points bellow each sleeper.

Rail roughness has been modeled by considering a profile consistent with Braun & Hellenbroich spectral density model [5]:

$$G_{rr}(n) = G_{rr}(n_0)\left(\frac{n}{n_0}\right)^{-\omega} \qquad (1)$$

where $n_0=1/2\pi$, and $\omega=3.5$. From this profile, considering the train speed and the properties of the contact element, a moving self equilibrated couple of reaction forces can be defined. This couple is considered in a second time history analysis, allowing new reaction force histories to be obtained. Since this contribution to reaction force is independent on axle load, only time lags need to be considered when building the complete time history of reaction forces produced by rail roughness when the train passes. These values can be added to previous ones thus obtaining the total reaction force histories at each point of the ballast- subballast interface.

The second step is performed in the frequency domain considering transfer functions from force at each emitter point of the ballast-subballast interface and acceleration at the reference point. For each sleeper, two emitter points have been considered, one bellow each rail. It is assumed that each rail contributes only to its closer emitter point.

Transfer functions are computed by performing harmonic analysis with a finite element model of the culvert plus the embankment and the ground. For a given frequency, all transfer functions are calculated simultaneously taking advantage of the reciprocity principle. A unitary force is applied at the node which response is desired while displacements are measured at all source points considered. Due to the symmetry of the problem, only one half of the geometry is modeled.

The model mesh is presented in Figure 9. Five different materials have been defined: the soil, the embankment, the backfill, the ballast and the concrete. Elements in Figure 9 have been colored depending on their material. It contains 20 node solid elements, and dashpots in the boundaries properties of which are set to avoid reflection of incident waves. In some portions of the backfill, the mesh has been built with 10 node tetrahedra.

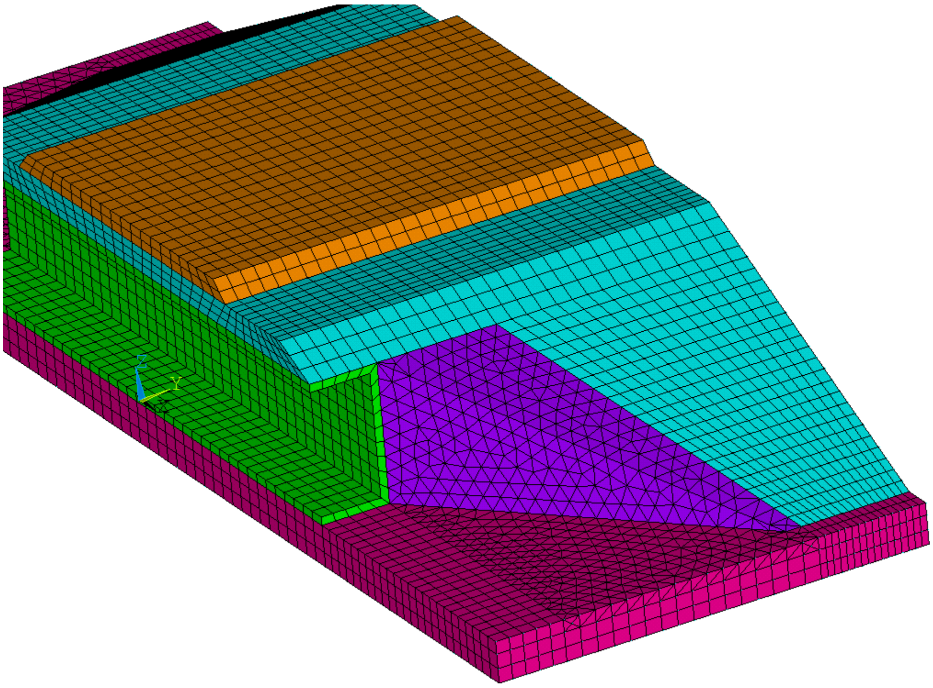

Figure 9. FE mesh used for obtaining transfer functions.

Element sizes have been set such that there are at least 4 elements per wavelength at 120Hz, considering the embankment as the softest element . The number of degrees of freedom of the model has been limited in order for the computer to deal with the problem using only RAM memory (the computer machine used in calculations has 6Gb). Otherwise computing time increases dramatically. The model extends 15.5m at both sides, perpendicular to the track axis, and 10.8m in its direction and has ~1 million degrees of freedom.

Using the two models described previously, simulations have been performed. Figure 10 shows a comparison between one set of recordings and simulations for an s-130 train with v=200km/h. Comparisons are performed using RMS values (1s windows). The reasons for using the latter value for comparisons are twofold. First, peak acceleration is highly dependent on the high frequency content of the signal, and the simulations have energy up to 120Hz . Secondly, peak values are usually linked to highly non linear phenomena such as shocks of wheels against the rail, and these are not modeled. The agreement is quite good although no adjustment of FEM parameters have been performed.

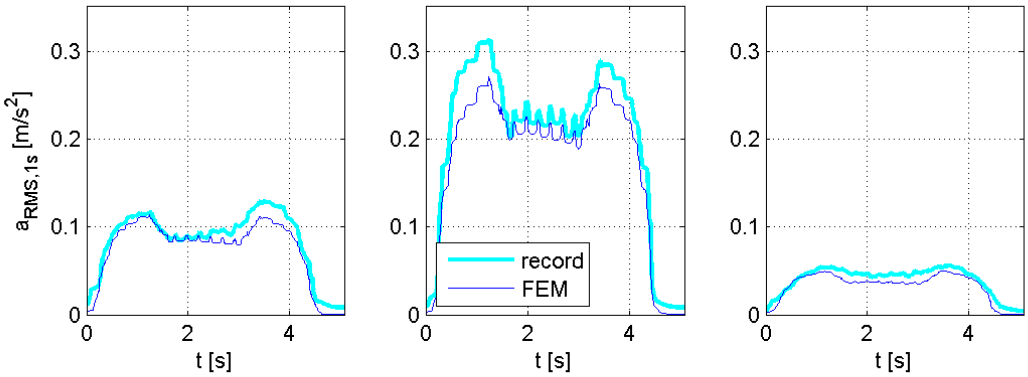

Figure 10. Comparison of recordings and simulations. Train s-130 with v=200km/h

More simulations have been performed in order to assess the response in points where no measurements were performed. Figure 11 shows a plot where measurements for s-130 trains at 200km/h are compared with predictions at mid-points of each pre-cast block. Predictions have been performed considering two different roughness levels, in order to identify an upper and a lower bound for the response. In this case, the lower bound is associated to the lowest roughness level (best quality) in the Braun and Hellenbroich model. The upper bound is associated with an intermediate roughness level.

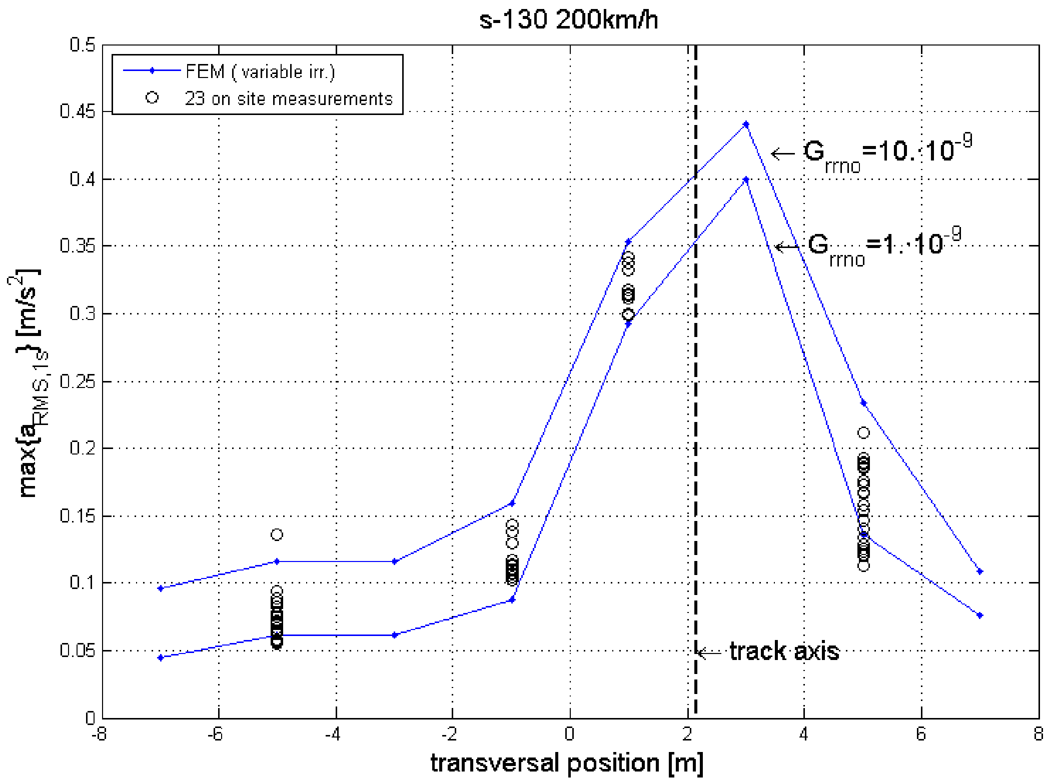

Figure 11. Simulations vs. measurements along the roof midline.

5. CONCLUSIONS

This work presents a set of measurements and models providing clues to understanding the dynamic behavior of frequently encountered underpasses.

Measurements have proved that acceleration scatter is quite important. Also, response distribution along the underpass axis has been found to be non-uniform, with the relative position between track axis and sensor being very important.

Finite element techniques have been useful to model the dynamic behavior of this structure, using a set of two models: one for the superstructure and another for the ensemble structure-embankment- soil. These models have allowed assessing a detailed distribution of response along the upper plane of the structure.

5. ACKNOWLEDGEMENTS

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